# GEOTECHNICAL ENGINEERING REPORT

## ACTIVITY 130 REPORT
## GEOTECHNICAL AND MATERIALS FOR PLAN-IN-HAND

## BELFRY - NORTH
## STPP 72-1(1)10 CN 1016

TERRACON PROJECT NO. 26015063
November 11, 2005

*Prepared for:*


David Evans & Associates, Inc.
West 110 Cataldo
Spokane, Washington 99201


*Prepared by:*


Terracon
2110 Overland Avenue, Suite 124
Billings, Montana 59102
Phone: 406-656-3072
Fax: 406-656-3578


MONTANA
WALTER J.
FEEGER
16659
11/11/05
LICENSED
PROFESSIONAL ENGINEER

Terracon

Terracon
2110 Overland Ave., Suite 124
Billings, MT 59102
(406) 656-3072 Fax: (406) 656-3578

November 11, 2005

Mr. Larry Olson, P.E.
David Evans & Associates, Inc.
West 110 Cataldo
Spokane, Washington 99201

**Subject:** **Activity 130 – Geotechnical Engineering and Materials Report For Plan-In-Hand**
**Belfry – North STPP 72-1(1)10 CN 1016**
**Belfry, Montana**
**Terracon Project No. 26015063**

Dear Mr. Olson:

Terracon has completed the activities associated with the Geotechnical Engineering and Materials for Plan-in-Hand for the Belfry North project. This report presents the results of our field exploration, laboratory testing and recommendations for pavement construction, earthwork, embankments, culverts, and bridge replacement.

We appreciate being of service to you in the geotechnical engineering phase of this project. If you have any questions concerning this report, please contact us at your convenience.

Sincerely,
**TERRACON**

Walter J. Feeger, P.E.
Geotechnical Department Manager

for
Daniel C. Nebel, P.G.
Office Manager

Copies to: Addressee (5)

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PLAN-IN-HAND

BELFRY – NORTH
STPP 72-1(1)10 CN 1016
BELFRY, MONTANA

TERRACON PROJECT NO. 26015063
NOVEMBER 11, 2005

## INTRODUCTION

This report presents the results of our geotechnical engineering exploration for the proposed Belfry North project which includes roadway reconstruction, realignment, culvert replacement, and bridge construction on Highway 72 in Belfry, Montana.

The purpose of the services associated with this Activity 130 report is to provide information and geotechnical engineering recommendations relative to:

- subsurface soil and groundwater conditions
- pavement design
- design of permanent cut and fill slopes
- culverts
- bridge design
- drainage
- earthwork

The recommendations contained in this report are based upon the results of field and laboratory testing, engineering analyses, and experience with similar soil conditions, similar structures and our understanding of the proposed project.

## PROPOSED CONSTRUCTION

As indicated, this project will consist of the reconstruction of a segment of Highway 72 extending northward from Belfry, Montana. The project begins at STA 10+00, about 185 meters south of the junction with Highway 308 and extends northward to the intersection with Highway 310 at STA 182+17.19, for a total project length of about 17.22 km. The project includes widening the road to a 10.4-meter width, along with several horizontal and vertical alignment changes within the rural segment.

Within the improvements adjacent to the town of Belfry and the Highway 310 intersection, the typical section is anticipated to include curb and gutter. Appendix A includes the Plan and Profile sheets, which conceptually details these improvements along with indicating the boring locations. Throughout most of the project length the horizontal alignment approximately parallels the existing alignment and generally less than 2 meters of vertical alignment change occur along the new centerline. The more significant horizontal alignment changes are summarized in the following table.

| Approx. STA Limits | Remarks |
|---|---|
| 11+77 to 35+60 | Follows old RR grade to bypass Belfry. |
| 71+00 to 82+40 | Remove a low speed curve. |
| 164+00 to 168+00 | Remove a low speed curve. |
| 176+00 to 182+17.19 | New Intersection with Hwy. 310 |

Structures are anticipated at four locations; one across Bear Creek near STA 13+90, two across the Clark's Fork of the Yellowstone River near STA 29+07 and STA 70+42, and one across Silver Tip Creek near STA 57+74.

Beginning near STA 167+00 and continuing to about STA 177+00, a steep hillside ascends along the west side of the proposed alignment. Anticipated cut backslope heights have not been determined at this time, but may approach heights on the order of 6 to 8 meters.

The Billings District Office of MDT performed a Soil Survey for this segment of Highway 72 in 1989. It was requested that Terracon use that information to the greatest extent practical in preparing both the Activity 106 and 130 reports. A copy of the MDT Soil Survey, dated November 16, 1989, and related documents are included in Appendix D.

Our Activity 106 field exploration work scope consisted of visual observations of surface conditions, performing confirmation soil survey borings at eight (8) locations along the alignment, obtaining samples for corrosion tests and documenting the conditions at seven (7) existing culverts. Laboratory testing was limited to verifying AASHTO classifications, moisture content, and moisture-density relationship. Additional R-value tests were not performed.

The Activity 130 field exploration work scope was comprised of performing twelve (12) borings at the proposed new bridge locations, nine (9) borings at the major culvert replacement locations, two (2) borings within the anticipated embankment fill footprint near the north end of the project, and two (2) borings in the area of the proposed rock cut at the north end of the project (one boring at the base of the slope and one boring at the top of the slope). Laboratory testing consisted of moisture content, Atterberg Limits and gradation analysis for AASHTO classifications, unconfined compression, consolidation/swell, and moisture-density relationship.

## SITE EXPLORATION

The scope of the services performed for this project included a site reconnaissance by a geotechnical engineer, a subsurface exploration program, laboratory testing and engineering analyses.

**Site Reconnaissance:** Terracon personnel performed a site reconnaissance prior to finalizing locations for subsurface explorations. A common site condition throughout most of the project length is that irrigated fields and pastures are usually present on one or both sides of the alignment. Local irrigation practices can cause large seasonal water content variations in the subgrade soils, especially in the immediate vicinity of irrigation ditches. The implication is that the potential exists for encountering soft and wet subgrade soils almost anywhere in the vicinity of irrigation ditches and irrigated fields or pastures.

The topography along the alignment from about STA 11+85 to 35+60 follows a former railroad alignment. That segment crosses mostly farm fields, skirts one edge of the Belfry sewage lagoons and crosses both Bear Creek and Clark's Fork of the Yellowstone River. The topography is mostly flat lying, with a very gentle slope down toward the north. The proposed finished grade closely follows the existing grade, except at the creek and river crossings, where the stream beds are incised about 5 to 6 meters below the surrounding topography.

From about STA 35+60 to about STA 72+00, the proposed alignment closely follows the existing alignment, shifting slightly to the east for a parallel bridge across Silver Tip Creek and Clark's Fork of the Yellowstone River. Topography within that segment is flat lying with irrigated pastures and farm fields generally located along both sides of the alignment.

From about STA 72+00 to STA 82+50, a shift in the horizontal alignment occurs to remove a low speed curve. The proposed alignment crosses an irrigated field and is typically less than 2 meters above the existing grade across the field.

From about STA 82+50 to STA 164+50, the alignment closely follows the existing alignment, typically offset less than about 10 meters east of the existing centerline. Land use on each side of the alignment is predominantly irrigated pasture and fields. The Clark's Fork of the Yellowstone River is very close to the edge of the right-of-way between about STA 157+00 to STA 161+00.

From about STA 164+50 to about STA 168+00 the proposed alignment shifts slightly west of existing centerline to improve sight distance and speed around a curve. This segment will require a fill having a maximum height of about 6 meters. It also begins a segment where steep slopes ascend from the west side of the alignment.

Beginning about STA 167+00 to STA 174+00, steep soil and rock slopes ascend along the west side of the alignment. The rock consists of the Eagle Sandstone and forms a very steep face that becomes nearly vertical. The soils along the base of the rock face are a combination of slope wash and colluvium, weathered from the rock. Cobble to boulder sized rocks litter the surface below the rock face and are probably also buried below the soil surface.

From about STA 168+00 to STA 181+00 the proposed alignment closely parallels the existing alignment. It traverses the lower portion of an undeveloped hillside that ascends west of the alignment. On the east side is a combination of pasture, fields, farm buildings, residences and small businesses.

The final alignment segment from about STA 181+00 to STA 182+17.19 deviates to the east of the existing alignment to make a new intersection with Highway 310. This short segment cuts across an open area used by an adjacent small business.

**Field Exploration:** A total of thirty-three (33) borings were drilled in conjunction with the Activity 106 and Activity 130 studies. Borings B-1 through B-8 were drilled in conjunction with the Activity 106 study in December of 2004. Borings B-9 through B-33 were drilled specifically for the Activity 130 study from August through October 2005. The sample information associated with the Activity 130 exploration has been appended to the end of Form 111 in Appendix B. The borings along the existing and proposed alignment were drilled to approximate depths ranging from about 1.7m to 19.6m at the locations shown on the enclosed Plan and Profile sheets in Appendix A. A 45.7 meter boring was advanced at the top of the proposed rock cut on the north end of the project. The borings were advanced using a truck-mounted drilling rig, equipped with hollow stem augers and wire line coring equipment.

Our field engineer recorded a log of each boring during the drilling operations. At selected intervals, samples of the subsurface materials were taken by pushing thin-walled Shelby tubes or by driving split-spoon samplers with a 140-pound automatic hammer. Where encountered, bedrock was cored below the overburden soils using HQ and NQ coring equipment.

Rock Quality Designation (RQD) values were assigned to the core samples of the bedrock retrieved from the respective borings. Bulk samples of subgrade materials were obtained from the auger cuttings. In addition, a measurement for groundwater was made upon completion of each boring.

The following table summarizes the location and purpose of each boring.

| Boring No. | Purpose | Depth, m | Location |
|---|---|---|---|
| B-1 | Activity 106 - Subgrade along new alignment | 3.5 | STA 20+40 |
| B-2 | Activity 106 - Subgrade along new alignment | 3.5 | STA 27+20 |
| B-3 | Activity 106 - Subgrade along new alignment | 1.7 | STA 36+20 |
| B-4 | Activity 106 - Subgrade along existing alignment | 1.8 | STA 55+10 |
| B-5 | Activity 106 - Subgrade along existing alignment | 3.5 | STA 82+90 |
| B-6 | Activity 106 - Subgrade along existing alignment | 3.5 | STA 127+70 |
| B-7 | Activity 106 - Subgrade along existing alignment | 3.5 | STA 163+60 |
| B-8 | Activity 106 - Subgrade along existing alignment | 3.5 | STA 181+80 |
| B-9 | Bear Creek Bridge abutment foundation | 19.6 | STA 13+70 |
| B-10 | Bear Creek Bridge abutment foundation | 18.3 | STA 14+04 |
| B-11 | Clark's Fork South Bridge abutment foundation | 13.7 | STA 28+50 |
| B-12 | Clark's Fork South Bridge bent foundation | 17.1 | STA 28+80 |
| B-13 | Clark's Fork South Bridge bent foundation | 15.2 | STA 29+20 |
| B-14 | Clark's Fork South Bridge abutment foundation | 10.7 | STA 29+60 |

| Boring No. | Purpose | Depth, m | Location |
|---|---|---|---|
| B-15 | Silver Tip Creek Bridge abutment foundation | 4.3 | STA 57+50 |
| B-16 | Silver Tip Creek Bridge abutment foundation | 4.3 | STA 57+90 |
| B-17 | Clark's Fork North Bridge abutment foundation | 10.7 | STA 69+90 |
| B-18 | Clark's Fork North Bridge bent foundation | 15.8 | STA 70+10 |
| B-19 | Clark's Fork North Bridge bent foundation | 16.8 | STA 70+47 |
| B-20 | Clark's Fork North Bridge abutment foundation | 10.7 | STA 70+70 |
| B-21 | Dry Creek Canal South culvert foundation | 5.9 | STA 72+06 |
| B-22 | Golden Ditch Canal culvert foundation | 6.2 | STA 86+84 |
| B-23 | Dry Creek Canal North culvert foundation | 6.2 | STA 103+80 |
| B-24 | Dubs Waste Ditch culvert foundation | 7.6 | STA 129+03 |
| B-25 | Graham Waste Ditch culvert foundation | 5.8 | STA 145+88 |
| B-26 | Fisher Waste Ditch culvert foundation | 6.2 | STA 151+80 |
| B-27 | BLM Drainage culvert foundation | 6.2 | STA 155+24 |
| B-28 | Sand Creek Canal South culvert foundation | 6.2 | STA 157+48 |
| B-29 | Subgrade and embankment along new alignment | 5.8 | STA 164+50 |
| B-30 | Subgrade and culvert foundations along existing alignment | 6.2 | STA 165+50 |
| B-31 | Sand Creek Canal North culvert foundation | 6.2 | STA 166+15 |
| B-32 | Rock Cut Slope (Top) | 45.7 | STA 167+10 |
| B-33 | Rock Cut Slope (Base) | 6.3 | STA 168+80 |

It is our understanding the MDT Soil Survey included about 21 borings along a similar alignment, with four other borings along a different alignment. The approximate locations of the borings performed by MDT were correlated to the project's metric stationing and elevations were estimated based on the metric contours. The approximate locations and logs of the MDT borings are shown on the Plan and Profile Sheets.

The existing base course classified as A-1-a and A-1-b. Two R-value tests reported by MDT indicated R-values of 77 and 79 for the base course.

The subgrade soils along the alignment varied among A-4, A-6 and A-7-6. The variations in the subgrade classifications occurred throughout the project length, so that it would not be practical to identify separate segments for different pavement sections.

The A-6 and A-7-6 subgrade soils therefore control the pavement section design. By default, the materials having AASHTO classifications of A-6 and A-7-6 are assigned an R-value of 5.

**Laboratory Testing:** The samples recovered during the field exploration were transported to the laboratory where the project geotechnical engineer visually classified them. A copy of the Unified Soil Classification System used to classify the samples is presented in Appendix C. The field descriptions were confirmed or modified as necessary and an applicable laboratory testing program was formulated to determine properties of the subsurface materials considered appropriate for this project. Boring logs were prepared and are presented in Appendix A. Simplified logs are presented on the Plan and Profile sheets, also in Appendix A.

Laboratory tests were conducted on selected samples and the results are presented on the logs, Form 111, Table of Soil Corrosion Test Results, or in Appendix B. The test results were used for the geotechnical engineering analyses, and for the development of the recommendations. Selected samples were tested for the following properties:

- Water Content
- Dry Density
- Swell/Consolidation
- pH and Marble pH
- Electrical Conductivity
- Moh's Hardness
- Grain Size Distribution
- Liquid and Plastic Limits
- Moisture-Density Relationship
- Unconfined Compression
- Water Soluble Sulfate Content

In addition, compressive strength tests were performed on samples of the claystone bedrock obtained from several of the borings at the proposed bridge locations and the sandstone bedrock from the proposed rock cut on the north end of the project. A summary of those results is included in the following table:

| Boring | Depth, m | Bedrock Type | Compressive Strength, MPa |
|---|---|---|---|
| B-11 | 13.4 | Claystone | 84.9 |
| B-13 | 14.3 | Claystone | 14.3 |
| B-16 | 16.0 | Claystone | 46.1 |
| B-20 | 10.4 | Claystone | 57.1 |
| B-32 | 21.8 | Sandstone | 13.8 |
| B-32 | 45.6 | Sandstone | 23.7 |

## SURFACE AND SUBSURFACE CONDITIONS

In preparation for this project, Terracon reviewed the previous MDT Soil Survey report dated November 16, 1989 and various geology reports and maps. The previous Soil Survey indicates the subgrade soils are predominantly A-4 and A-6 materials.

A review of the Geology Map of Montana (MGMB, 1955) indicates the alignment is almost entirely in alluvium of the Clark's Fork of the Yellowstone and its tributaries.

A review of the Geology and Mineral Resources of Parts of Carbon, Big Horn, Stillwater and Yellowstone Counties, Montana (United States Department of the Interior Geologic Survey Bulletin 822-A, dated 1930), indicates that the rock outcrops adjacent to the alignment segment from about STA 167+00 to 177+00 belong the Eagle Sandstone.

**Groundwater Conditions:** Groundwater accumulated in a majority of the borings during the time they were open. Groundwater was measured at depths ranging from approximately 1.5m to 7m below existing grades. The depth or lack of groundwater is noted on each respective boring log in Appendix A. Where clay soils were encountered, it is likely the clay content prevented water from accumulating in the borings. These observations represent groundwater conditions at the time of the field exploration, and may not be indicative of other times, or at other locations. Groundwater can be expected to fluctuate with varying seasonal, and weather conditions as well as with irrigation practices. It should be noted that during the irrigation season, groundwater may be at or near the surface at many locations.

## CULVERT CONDITION REPORT

David Evans & Associates provided a listing of culverts along the existing alignment. Locations for culverts along the new alignment segments had not yet been identified at the time of this report.

Based on the listing provided, seven (7) existing culvert locations were selected for observation and to obtain samples for corrosion tests. The culvert condition report is contained in Appendix B, along with a table presenting results of the corrosion tests. A copy of the Culvert Condition Report performed by MDT in 1990 is also included n Appendix B.

The culvert condition reports indicate a wide variety of pipe conditions. Typically, the larger culverts tended to be in fair to good condition, while the smaller ones tended to suffer from rust and sedimentation blockage. The results of the corrosion tests indicate that the soils are typically not corrosive to steel, zinc, aluminum, or concrete; however an occasional test result does indicate corrosiveness to steel and zinc.

## ENGINEERING ANALYSIS AND RECOMMENDATIONS

**Geotechnical Considerations:** The primary geotechnical consideration associated with the project will be providing foundation support for the replacement bridges. The preliminary plan and profile sheets provided by David Evans & Associates indicate the grade increase for the abutment approaches will be about 1 to 3 meters. This range of additional fill height will likely produce negligible settlement in the natural materials below the approach fill provided the subgrade surfaces are properly prepared prior to fill placement. The locally available materials are predominantly coarse-grained granular materials, which are good for raising the bridge approaches.

The subsurface conditions encountered are favorable for supporting the proposed bridges using steel H-piles, and/or drilled piers. It should be noted that for both of these alternatives, the presence of potentially large cobbles/boulders and groundwater may complicate the foundation construction process.

**Drilled Pier Foundation System:** Based on information provided by David Evans & Associates, it is our understanding that the use of drilled pier foundations, bearing in bedrock, are desired at the intermediate bridge bent locations. For drilled piers, either cased holes and/or a slurry drilling technique will be required to maintain an open hole. Special drilling tools, such as boulder extractors, hard rock core barrels, calix and/or multi-roller rock bits will probably be required to penetrate to the design pier depth.

Drilled pier contractors should be prepared to drill through hard cobble/boulder sized material as well as cemented sandstone bedrock. The most difficult drilling condition will probably occur when a large cobble/boulder is encountered across only part of the pier diameter.

Dewatering the piers for inspection may not be possible because the resulting hydrostatic imbalance could create a quick condition at the pier base. As a result, concrete must be placed using a pump truck or a tremie pipe, with the discharge kept below the top of the fresh concrete to displace water or drilling slurry. Casing should be withdrawn in a slow continuous manner while maintaining a sufficient head of concrete to prevent infiltration of water and slough to prevent creation of voids in the pier concrete.

Drilled piers achieve intimate contact with the surrounding materials and thereby transmit the load through a combination of shaft friction and end bearing. In the gravel and claystone, a high degree of shaft friction will be achieved.

Drilled piers bearing in the un-weathered claystone bedrock at each bridge location will develop their capacity through a combination of end bearing and friction along the shaft length.

For vertical capacity, the drilled piers should be designed using an allowable bearing capacity of 1,437 $kN/m^2$ when founded at least 1.5 m into un-weathered claystone bedrock. A skin friction value of 71.9 $kN/m^2$ may be used for the portion of the pier in the un-weathered claystone bedrock. The allowable end bearing pressure includes a safety factor equal to 3.0. In our opinion, the smallest practical pier diameter that should be attempted in these deposits is 1219 mm.

Potential uplift loads should be calculated using 15 percent of the expansive pressure. Based on the laboratory tests, the following potential uplift loads could be realized in the claystone bedrock:

| Bridge Location | Potential Uplift Loads, kPa |
|---|---|
| Clark's Fork South | 22.3 |
| Clark's Fork North | 15.8 |

For estimating drilled pier lengths at the intermediate bridge bent locations, we recommend the following approximate un-weathered claystone contact elevations be used.

| Bridge Location | Anticipated un-weathered Claystone contact elevation, meters** |
|---|---|
| Clark's Fork South – South Bent | 1143 |
| Clark's Fork South – North Bent | 1147 |
| Clark's Fork North – South Bent | 1142 |
| Clark's Fork North - North Bent | 1141 |

**Please note that these elevations are for estimation purposes only. As variations in the contact elevation can vary significantly from one location to another, the final pier length will need to be determined in the field by qualified personnel.

**Driven Steel H-Pile Foundation System:** Based on information provided by David Evans & Associates, it is our understanding that steel H-piles, driven to practical refusal in the bedrock, are desired for the bridge abutment foundations. In general, the soil profile at the proposed bridge abutment locations consists mainly of sand and gravel soils overlying claystone bedrock. Steel H-piles are the most practical driven pile option for these subsurface conditions. The H-piles should be driven through the gravel and reach practical refusal in the un-weathered claystone bedrock. When driven to practical refusal in the claystone, the compressive capacity of steel H-piles is governed by the allowable stress in the steel, 62 MPa. Very little friction will be developed in the cohesionless soils encountered above the bedrock. Our experience indicates practical refusal will generally occur within about 1.5 to 3 meters below the un-weathered claystone contact elevation. For estimating pile lengths, we recommend the following approximate un-weathered claystone contact elevations be used.

| Bridge Location | Anticipated un-weathered Claystone contact elevation, meters** |
|---|---|
| Bear Creek | 1158 |
| Clark's Fork South | 1150 |
| Silver Tip Creek | 1139 |
| Clark's Fork North | 1139 |

**Please note that these elevations are for estimation purposes only. As variations in the contact elevation can vary significantly from one location to another, the final pile length will need to be determined in the field by qualified personnel.

The piles should be fitted with driving points to prevent damage while penetrating the gravel.

The term "practical refusal" must be determined by performing a wave equation analysis after the pile section is selected and details of the driving equipment are known.
After the contractor has selected the driving equipment, a wave equation analysis should be performed to develop the initial driving criteria. It is important that the first three piles driven at one abutment be monitored during installation using a dynamic analyzer to finalize the driving criteria. If piles are also selected for the intermediate bridge supports, a similar number of tests using a dynamic analyzer should be performed to finalize the driving criteria for those piles as well. Different driving criteria may apply at the abutments and intermediate supports due to differences in the pile lengths and whether or not the abutment locations are pre-bored.

At this time, and based on the preliminary information provided, it is our understanding that HP360x174 piles will likely be used at the bridge abutments. The allowable compression and uplift capacities at each bridge location for this pile section as well as other commonly used pile sections are presented in the following table. These capacities are based on the piles being driven to practical refusal in the claystone bedrock, so that the allowable stress in the steel controls the design for compressive load.

Pile groups should be designed with a center to center pile spacing of at least 3 pile widths.

| Bridge Location | Pile Section | Allowable Capacity, kN | |
|---|---|---|---|
| | | Compression | Uplift |
| Bear Creek | HP310 x 110 | 873 | 243 |
| | HP360 x 108 | 1045 | 281 |
| | HP360 x 174 | 1377 | 290 |
| Clark's Fork South | HP310 x 110 | 873 | 35 |
| | HP360 x 108 | 1045 | 41 |
| | HP360 x 174 | 1377 | 42 |
| Silver Tip Creek | HP310 x 110 | 873 | 170 |
| | HP360 x 108 | 1045 | 197 |
| | HP360 x 174 | 1377 | 203 |
| Clark's Fork North | HP310 x 110 | 873 | 41 |
| | HP360 x 108 | 1045 | 48 |
| | HP360 x 174 | 1377 | 49 |

Calculations for estimating pile settlement indicate that under the maximum recommended compression capacity, pile settlement should be about 13 mm or less. Criteria are presented in a following section for determining the lateral capacity of piles and piers.

**Lateral Capacity of Piles and Piers:** The material properties presented below should be used for determining the lateral capacity of piles and piers. These values represent our best estimate of the actual value for each material property; no safety factor is included. An appropriate safety factor should be applied to the final calculated value for lateral capacity, or allowable deflection.

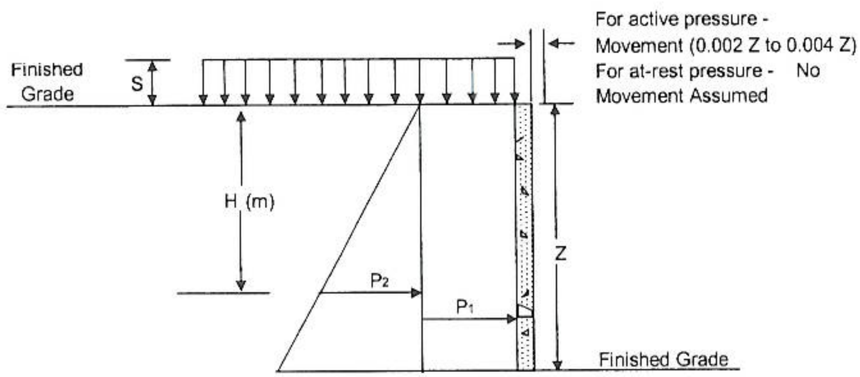
It is common practice to ignore the contribution of the upper 1.5 meters of soil when calculating the lateral capacity. Additionally, for the intermediate supports, we recommend groundwater be assumed at the ground surface. Depending upon the channel armoring provided, it might also be necessary to analyze the lateral pier capacity under the scenario of maximum scour. The constant of horizontal subgrade reaction increases linearly with depth.

| CRITERIA FOR LATERAL PILE AND PIER CAPACITY | | | |
|---|---|---|---|
| Material Property | Recommended Value | | |
| | Natural Clay & Sand | Natural Gravel | Abutment Fills |
| Moist Unit Weight | 18.1 kN/m$^3$ | 21.2 kN/m$^3$ | 18.8 kN/m$^3$ |
| Saturated Unit Weight | 18.8 kN/m$^3$ | 22.8 kN/m$^3$ | -- |
| Submerged Unit Weight | 9.0 kN/m$^3$ | 13.0 kN/m$^3$ | -- |
| Cohesion | 24 kPa | 0 kPa | 0 kPa |
| Internal Friction Angle | 24° | 36° | 32° |
| Horizontal Subgrade Modulus Note 1, 2 & 3 | $n_h$ = 2,700 kPa/m | $n_h$ = 27,000 kPa/m | $k_h$ = 1,600 kPa/B |

Note 1: $n_h$ increases linearly with depth.
Note 2: B = pile or pier diameter in meters.
Note 3: $k_h$ remains constant with depth.

**Lateral Earth Pressures:** If the bridge abutments are designed like a retaining wall such that there are unbalanced backfill levels on opposite sides then they should be designed for earth pressures at least equal to those indicated in the following table. Earth pressures will be influenced by structural design of the abutments, conditions of abutment restraint, methods of construction and/or compaction and the strength of the materials being restrained.

Two abutment restraint conditions are shown. Active earth pressure is commonly used for design of free-standing cantilevered abutments and assumes abutment movement. The "at-rest" condition assumes no abutment rotation. The recommended design lateral earth pressures do not include a factor of safety.




## EARTH PRESSURE COEFFICIENTS

| EARTH PRESSURE CONDITIONS | COEFFICIENT FOR BACKFILL TYPE | EQUIVALENT FLUID PRESSURE ($kN/m^3$) | SURCHARGE PRESSURE, $P_1$ (kPa) | EARTH PRESSURE, $P_2$ (kPa) |
|---|---|---|---|---|
| Active (Ka) | Granular - 0.30 | 5.5 | (0.30)S | (5.5)H |
| At-Rest (Ko) | Granular - 0.45 | 8.7 | (0.45)S | (8.7)H |
| Passive (Kp) | Granular - 3.3 | 81.7 | --- | --- |

Conditions applicable to the above conditions include:

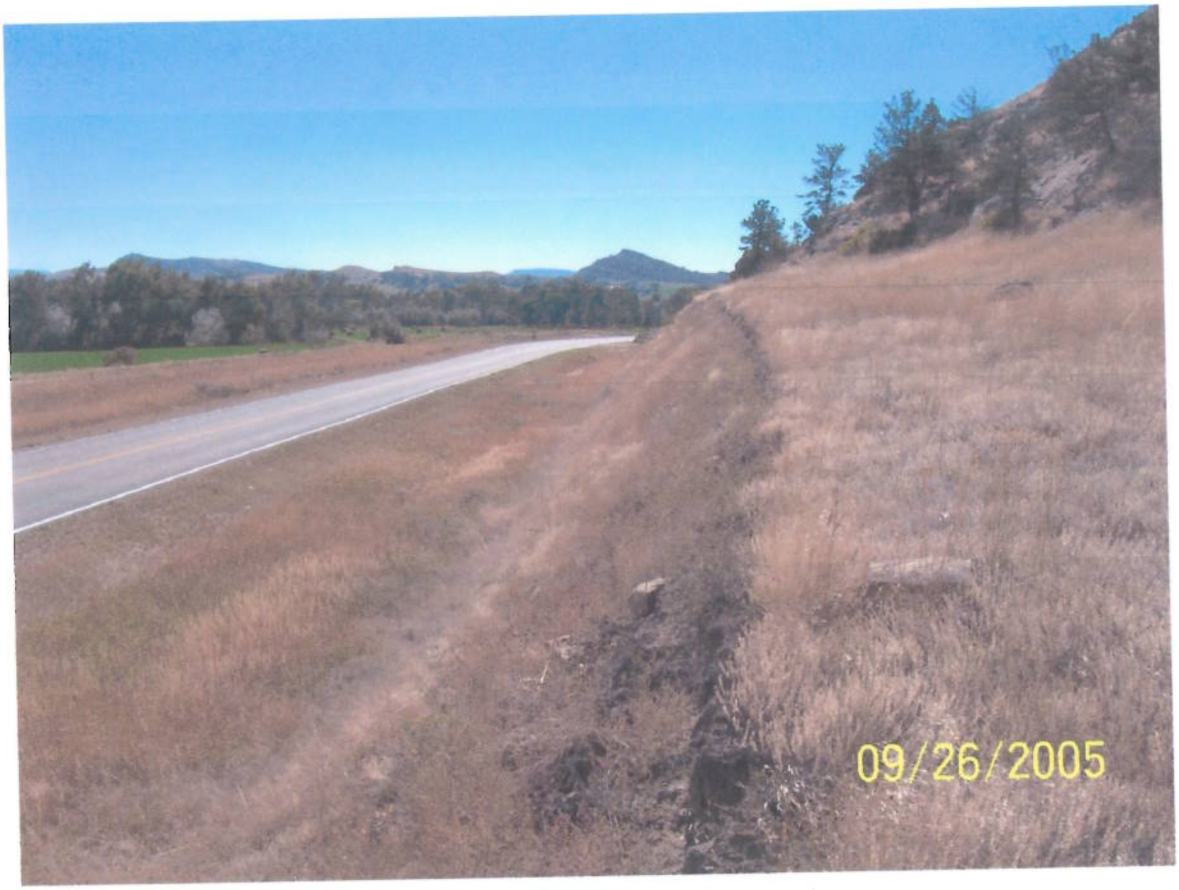
- For active earth pressure, abutment must rotate about base, with top lateral movements 0.002 Z to 0.004 Z, where Z is abutment height
- For passive earth pressure, wall must move horizontally to mobilize resistance.
- Uniform surcharge, where S is surcharge pressure
- In-situ soil backfill weight a maximum dry density of 21.2 $kN/m^3$
- Horizontal backfill, compacted to at least 95% of MT-210
- Loading from heavy compaction equipment not included
- No groundwater acting on wall
- No safety factor included

Backfill placed against abutments should consist of granular soils, similar to the on-site soils, but with particles greater than 75mm removed. For the granular values to be valid, the granular backfill must extend out from the base of the wall at an angle of at least 45 and 60 degrees from vertical for the active and passive cases, respectively.

To control the water level behind the abutments, we recommend weep holes be installed at close spacings across the abutment face. If this is not possible, then combined hydrostatic and lateral earth pressures should be calculated and added to the lateral earth pressure. These pressures do not include the influence of surcharges. Heavy equipment should not operate within a distance closer than the exposed height of the abutments to prevent lateral pressures more than those provided.

**Permanent Cut and Fill Slopes:** Based on a review of the Plan and Profile sheets, with the exception of the proposed cut slopes on the north end of the project, approximately between Stations 167+00 and 174+00, it appears that cuts will typically be less than about 1m to 2m. Most of the alignment is expected to have an increase in the centerline elevation. At most locations, the amount of fill is anticipated to be less than about 2m to 3m, although some exceptions occur. Permanent cut slopes should be planned no steeper than 2H:1V and permanent fill slopes and embankments should be planned no steeper than 3H:1V. At those maximum slopes, the cuts and fill are anticipated to be stable. Flatter slopes may be desirable for establishing vegetation and limiting erosion.

**Proposed Rock Cut Slope at North End of Project:** Between about stations 167+00 and 174+00, the proposed alignment consists of the raising and shifting of the roadway to the west and into the existing cut slope.




**Looking south along existing cut slope at about Station 169**

Based on the soils and bedrock encountered in Borings B-32 and B-33 and a visual review of the slope face and geologic mapping of outcrops the following materials are expected along the face of the existing slope face.

| | |
|---|---|
| Stations 167+00 to 167+30 | Slope wash with potential for bedrock within a few meters into slope. The existing cut face is within 10 to 20 meters of cliff face. |
| Stations 167+30 to 168+30 | Bedrock outcrop |


**Bedrock in cut slope at about Station 167+40**

| | |
|---|---|
| Stations 168+30 to 168+80 | Slope wash with potential for bedrock within a few meters into slope. The existing cut face is within 10 to 20 meters of cliff face. |
| Stations 168+80 to 171+00 | Slope wash with less potential for bedrock within 5 to 7 meters into slope. The cliff face is 20 meters or more from existing cut face. |
| Stations 171+00 to 173+50 | Slope wash with potential for bedrock within a few meters into slope. The existing cut face is within 10 to 20 meters of cliff face. |


**Looking west at cut slope and scarp face at about Station 172+60**

| | |
|---|---|
| Stations 173+50 to 174+00 | Slope wash with less potential for bedrock within 5 to 7 meters into slope. The cliff face is 20 meters or more from existing cut face. |

Depending upon the final slope configurations, additional geotechnical borings or shallow test pits may be required to provide an indication of the depth to bedrock in areas of slope wash. Without additional geotechnical borings or shallow test pits, it is impossible to ascertain whether and where bedrock will be encountered in cuts into the slope wash.

Slope Wash

The slope wash consists of a heterogeneous mixture of clay through boulder size materials formed at the base of a sandstone escarpment. Typically, the slope wash can be described as a sandy lean clay to sandy silt with scattered sandstone fragments.
The existing cut slope is stable with minor raveling. The existing cut slope was measured at about 80 to 85 percent from Stations 167+00 to 169+60 and up to about 6 meters high.

From about 169+60 to 174+00, the existing cut slope was measured at about 50 percent and up to about 4 meters high.

Based on the performance of the existing slopes, retaining structures should be anticipated for slopes formed in slope wash steeper than about 80 percent (1.25:1 H:V).

Bedrock

The bedock within the area of the existing cut slope is light brownish-gray cross bedded sandstone of the Cretaceous Eagle Formation. Three sandstone intervals 10 to 50 feet thick with intervening sandy shales as thick as 50 feet form the face of the escarpment.

Bedrock dip was measured at about 7 degrees to the west. Two prominent fracture planes are evident within the bedrock forming the escarpment. The dominant fracture plane trends north-south and dips at about 60 to 75 degrees east. The cliff face has formed along this fracture plane.


**Fracture face scarp at Station 167+20**

The other prominent fracture plane is oriented east-west, perpendicular to the north-south fracture plane, and dips to the south at about 70 to 90 degrees. Large blocks of sandstone form along the two fracture planes.

The north-south cliff forming joint set appears to be the controlling geologic factor with regard to slope angle for rock cuts at this location. It would appear that the final slope should closely approximate the controlling joint set dip angle of about 75 degrees (0.25:1 H:V) to vertical. The overburden soils and weathered rock near the top of the rock cut will need to be stabilized to prevent raveling of the slope.

Rockfall

Sandstone blocks are present within the slope wash along the cut face and on the surface of the slope wash extending away from the cliff face. Individual blocks were measured up to 30 meters from the cliff face. The bedrock blocks appear to the be the result of toppling of blocks that have formed along the two dominant fracture planes on the upper slopes.

General Recommendations

The following general recommendations for cut slopes at the north end of the project are provided.

1. Moving the roadway and shoulder closer to the cliff face will result in a greater threat of large sandstone blocks toppling onto the highway.

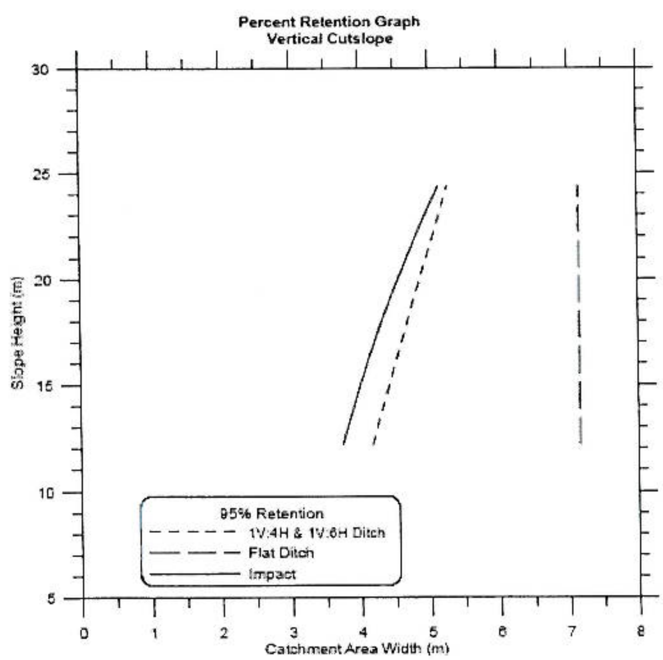
2. Cut slopes in slope wash up to about 7 meters high should be designed at a maximum of about 80 percent (1.25:1 H:V) to reduce slope instability and consequent raveling. A catchment area should be provided at the base of the slopes to prevent material from rolling onto the roadway. Shallow retaining structures could be incorporated into the base of the slopes to reduce final slope angles. Retaining structures will be required for cuts steeper than about 80 percent in slope wash.

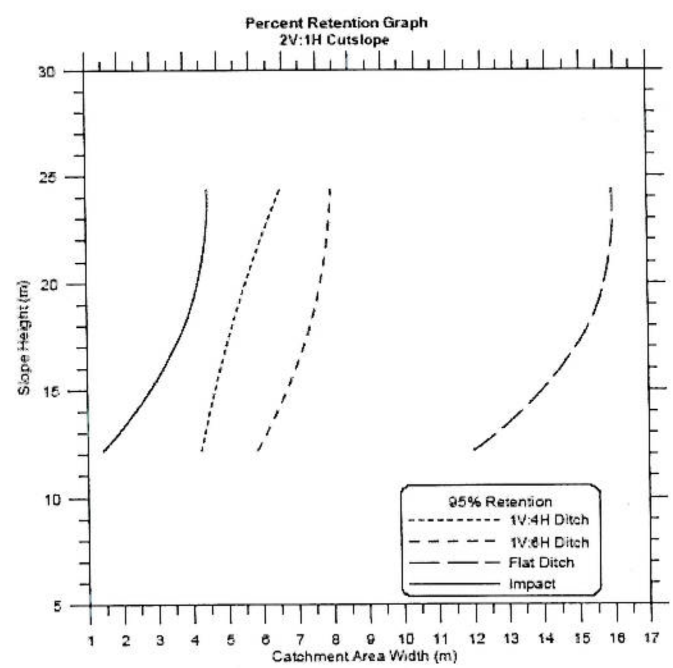
3. Cut slopes in bedrock should be at or near vertical to coincide with the dominant fracture patterns. Shallower rock slopes may serve to launch rock fragments onto the roadway.

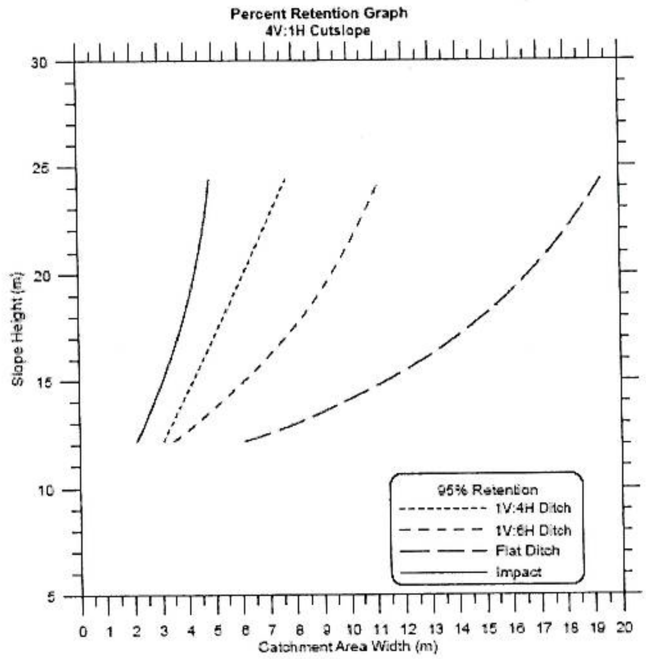
4. A rock catchment area should be provided at the base of rock cuts to reduce the potential for rocks to roll onto the pavement. Based on a slope height of 12 meters, the rock catchment ditch should be about 3 to 6 meters wide, depending on the grade of the cut slope, and be sloped at a 4:1 or 6:1 (H:V) grade.
   If a flat catchment ditch is used, the width would be about 6 to 12 meters depending on the grade of the cut slope and should be 1 meter deep.

A bench at the top of the rock cut to catch debris from the above rock cut should also be constructed. The constructed slope should be pre-split, scaled and cleaned to remove protruding or overhanging rocks. Joint orientation and weathering should be observed during construction by a geologist or geotechnical engineer experienced in rock mechanics to determine if the proposed slope design is adequate. (Reference: "Rockfall Catchment Design Area Design Guide", SPR-3 (032)).

A copy of the graphs used in preparing the recommended rock cut design is shown below.






**Culverts:** As requested, soil borings were performed at the following proposed culvert locations:

| Station | Location | Culvert Type |
|---|---|---|
| 72+06 | Dry Creek Canal South | 3600mm x 1800mm RCB |
| 86+84 | Golden Ditch | Unspecified RCB |
| 103+80 | Dry Creek Canal North | 3000mm x 1800mm RCB |
| 129+03 | Dubs Waste Ditch | 2700mm x 1800mm RCB |
| 145+88 | Graham Waste Ditch | 914mm RCP |
| 150+01 | Fisher Waste Ditch | 3050mm x 1830mm RCB |
| 155+24 | BLM Canyon | 2100mm x 1500mm RCB |
| 157+48 | Sand Creek Canal South | 3600mm x 2100mm RCB |
| 166+15 | Sand Creek Canal North | 3600mm x 2700mm RCB |

Based on a review of the Preliminary Plan and Profile Sheets, the concrete box culverts, with the exception of the Dry Creek Canal North and the Dubs Waste Ditch will likely encounter soft, wet clay or loose, wet sand foundation soil conditions at the channel bottom elevation.

To provide a solid working platform on which to set and backfill the culverts, we recommend the soft clay/loose sand soils be subexcavated to a depth of at least 0.6m and a geogrid or geotextile separation/stabilization fabric placed at the base of the subexcavation.

Granular foundation material should then be placed to the base of the desired bedding elevation. Alternatively, a bed of lean concrete would also be appropriate in lieu of the geogrid/geofabric and subexcavation combination. Inlet/outlet wingwalls at these locations should also be placed on the combination of geogrids or a geotextile separator with about 0.6 m of subexcavation and replacement with granular material.

Depending on the time of year when excavation work takes place, these working platforms would likely be substantially or entirely below the groundwater elevation. Scheduling culvert installations during the Spring before the irrigation ditches are in use is recommended to help reduce the severity of wet soil conditions. Nevertheless, contractors working on the project should anticipate de-watering the excavations and have equipment on-site that will lower and maintain the groundwater level below the base of the excavations.

Based on the Plan Detail provided for the Dry Creek Canal North Culvert, the anticipated flow line elevation will be at elevation 1142.35. Therefore, the culvert will likely be situated within the stiff lean clay soils. Additionally, it is anticipated that the Dubs Waste Ditch will also be situated within stiff lean clay soils. At this time, it does not appear that any special culvert foundation preparation will be required at these two sites. Likewise, at this time, it is anticipated that the clay soils should provide adequate support for culvert wingwalls. However, if soft, wet soils are encountered within the excavations, subexcavation, placement of a geogrid/geofabric, and replacement with granular fill will be required.

Where the design of the culverts includes restrained elements, such as wingwalls, the following soil design parameters should be used:

| Soil Type | Dry Unit Weight, $kN/m^3$ | Coefficient of Friction | At-Rest Earth Pressure, $kN/m^3$ |
|---|---|---|---|
| Clay | 18.1 | 0.35 | 13.3 |
| Sand | 19.6 | 0.45 | 10.2 |
| Gravel | 21.7 | 0.55 | 8.6 |

Please note that the at-rest lateral earth pressures do not include any factor of safety and are not applicable for submerged soils/hydrostatic loading. Additional recommendations may be necessary if submerged conditions are to be included in the design.

We recommend the culverts be designed and constructed following the guidelines specified under Section 17 "Drainage and Irrigation Design" of the Montana Department of Transportation (MDT) Montana Road Design Manual. Additionally, foundation and bedding materials should conform to the specifications as outlined in the MDT Standard Specifications for Road and Bridge Construction under Section 701.04 "Foundation and Bedding Materials for Structures".

Results of the corrosion tests performed in conjunction with Activity 106 are presented on tables in Appendix B. Those test results indicated that the soils along the alignment are typically not corrosive to steel, zinc, aluminum, or concrete; however an occasional test result did indicate corrosiveness to steel and zinc.

**Embankment Settlement:** Between about STA 164+30 and STA 166+80, the deepest portion of the proposed roadway embankment would extend about 2m to 6.5m above the surrounding grades. The upperlying soils within this segment of the alignment mainly consists of moderately compressible clay and sand overlying dense gravel, which has relatively low compressibility.

Our calculations and experience indicate that for the proposed embankment heights and cross sections, centerline settlement is likely to be in the range of approximately 0.01m to 0.16m. More than half of that amount would probably occur before placement of the asphalt surfacing.

## PAVEMENT SECTIONS

The average daily design traffic intensity, provided by MDT, consists of 119 ESAL. That value equates to a cumulative ESAL of about 869,000 over an analysis period of 20 years. The controlling subgrade consists of A-6 soils. By default, those materials are assigned an R-value of 5 for pavement design purposes.

Three pavement reconstruction concepts were considered for this project:

- Asphalt surfacing with a layer each of cement treated base and crushed base course;
- Asphalt surfacing with cement treated base; and
- Asphalt surfacing with crushed base course.

Pavement section alternatives were prepared following the procedures presented in the 1993 AASHTO Guide for Design of Pavement Structures.

Input variables used in the calculations are presented in the following table.

| Parameter | Value | Parameter | Value |
|---|---|---|---|
| Design R-value for A-6 Subgrade | 5 | Reliability, percent | 85 |
| Effective Resilient Modulus, kPa | 21,000 | Standard Deviation | 0.45 |
| Initial Serviceability Index | 4.2 | Terminal Serviceability Index | 2.0 |

The material coefficients used in the calculations are presented in the following table.

| Material | Structural Coefficient, per mm | Drainage Coefficient |
|---|---|---|
| Plant Mix Asphaltic Concrete, Grade S | 0.0130 | 1 |
| Cement Treated Base | 0.0079 | 1 |
| Crushed Aggregate Base Course, Type A, Grade 6 | 0.0047 | 0.9 |

The following tables present the pavement section alternatives described above. Each table is for the indicated design ESAL. A sample calculation is presented in the Appendix C.

| Cumulative Design Lane ESAL = 869,000 | | | |
|---|---|---|---|
| Pavement Option | Thickness of Pavement Section Materials, mm | | |
| | Plant Mix Surfacing, Grade S | Cement Treated Base | Crushed Aggregate Base Course, Type A, Grade 6 |
| Option 1: AC+CTB+CBC | 90 | 290 | 200 |
| Option 2: AC+CTB | 90 | 390 | -- |
| Option 3: AC+CBC | 90 | -- | 735 |

David Evans & Associates, Inc. personnel will perform economic analyses for each of the three options and submit those estimates under separate cover.

## EARTHWORK

**Site Preparation:** Strip and remove existing vegetation, debris and any other deleterious materials from proposed embankment locations.

Wherever existing slopes that are steeper than 5:1 (H:V) will be covered by fill, the existing slope should be benched. Benches should be wide enough to accommodate compaction and earth moving equipment, and to allow placement of horizontal lifts of fill.

Areas that will receive fill should be scarified to a minimum depth of 200 mm, conditioned to near optimum water content, and compacted to at least 95 percent of the maximum dry density determined by MT-210.

Evidence of seeps or unusually soft near surface (1m or less) conditions were not detected at locations where significant excavations or embankments are anticipated. We, therefore, do not anticipate dig outs will be necessary for embankment construction. However, depending on the time of year that construction takes place, soft surface soils may be encountered within the proposed embankment footprint due to localized irrigation practices. If soft soils are encountered, subexcavation, placement of a geogrid/geofabric, and replacement with granular fill will be required.

Groundwater should be anticipated in excavations for new and replacement culverts. To reduce the potential for encountering groundwater, culvert construction should be scheduled for the early spring before irrigation begins. If groundwater is encountered, it should be removed using a dewatering system that will lower and maintain the groundwater at least 0.6m below the excavation bottom.

Based on the subsurface conditions encountered by the geotechnical exploration, subgrade soils exposed during construction are anticipated to be relatively stable. However, precipitation, repetitive construction traffic or other factors may affect the stability of the subgrade. If unstable conditions develop, workability may be improved by scarifying and drying or by overexcavation of wet zones and stabilization with geotextile and granular material. For the overexcavation option, a layer of stabilization geotextile should be placed on the tender subgrade after removal of disturbed material. The geotextile should be covered with granular material up to the design grade, or at least 0.6m above the geotextile, whichever is less.

The individual contractor(s) is responsible for designing and constructing stable, temporary excavations as required to maintain stability of both the excavation sides and bottom. Excavations should be sloped or shored in the interest of safety following local and federal regulations, including current OSHA excavation and trench safety standards.

**Subgrade Preparation:** Subgrade soils should be scarified, moisture conditioned and compacted to a minimum depth of 200 mm. Subgrade should be compacted to at least 95 percent of the maximum dry density determined by MT-210. The water content and compaction of subgrade soils should be maintained until pavement construction.

**Fill Materials and Placement:** Clean (vegetation and debris free) on-site soils or approved imported materials may be used as fill material for embankment construction. Since imported material will likely be furnished by the contractor from MDT approved sources, no specific acceptance criteria is presented in this report for general embankment construction.

Embankment fill should be placed and compacted in horizontal lifts, using equipment and procedures that will produce recommended water contents and densities throughout the lift. Recommended compaction criteria for fill materials are at least 95 percent of the maximum dry density determined by MT-230 for A-1-a and A-1-b materials and at least 95 percent of the maximum dry density determined by MT-210 for all other materials.

## SHRINK - SWELL

Review of the proposed profile, indicates most of the cut along the project length will occur in the ditches adjacent to the alignment. An exception to that occurs from about STA 167 to 175, where the backslopes in a sidehill cut will generate a substantial quantity of cut. Part of the cut will be in sandstone.

The soils encountered along the alignment are generally expected to shrink from cut to fill, while the rock will exhibit a swell from cut to fill. The following two tables summarize the estimated shrinkage of soils based on the results of the in-place density tests and maximum dry density from laboratory compaction tests. The percentage shrinkage is shown for 100 percent and 95 percent of maximum dry density. In reviewing the MDT soil survey report, we identified a typographical error in the table of estimated soil shrinkage values. That error has been corrected in the table of MDT test data.

| Terracon Borings | | | | | |
|---|---|---|---|---|---|
| Boring @ STA | Maximum Dry Density, kN/m$^3$ | In-place Dry Density, kN/m$^3$ | Soil Type | Percent Shrinkage @ 100% $\Gamma_{d\,max}$ | Percent Shrinkage @ 95% $\Gamma_{d\,max}$ |
| 20+40 | 16.7 | 17.1 | A-6 | 2 (Swell) | 7 (Swell) |
| 27+20 | 17.4 | 15.8 | A-6 | 9 | 5 |
| 36+20 | 22.4 | 19.7 | A-1-b | 12 | 7 |
| 55+10 | 18.9 | 15.2 | A-4 | 20 | 15 |
| 82+90 | 20.8 | 16 | A-2-4 | 23 | 19 |
| 127+70 | 18.1 | 14.4 | A-4 | 21 | 16 |
| 163+60 | 18.9 | 15.4 | A-4 | 19 | 14 |
| 181+80 | 18.1 | 14.9 | A-6 | 18 | 13 |

| MDT Borings | | | | | |
|---|---|---|---|---|---|
| Sample @ STA | Maximum Dry Density, kN/m$^3$ | In-place Dry Density, kN/m$^3$ | Soil Type | Percent Shrinkage @ 100% $\Gamma_{d\,max}$ | Percent Shrinkage @ 95% $\Gamma_{d\,max}$ |
| 23+00 | 16.7 | 14.7 | A-7-6 | 12 | 7 |
| 189+00 | 17.8 | 14.3 | A-6 | 19 | 15 |
| 215.30 | 18.1 | 14.4 | A-4 | 20 | 16 |
| 242+75 | 18.3 | 15.3 | A-4 | 17 | 12 |
| 442+10 | 17.4 | 13.9 | A-6 | 20 | 16 |
| 451+00 | 17.8 | 13.2 | A-4 | 26 | 22 |
| 519+80 | 18.4 | 14.8 | A-4 | 20 | 15 |

Based on the information in the above table, a typical estimated shrinkage for the soils appears to be about 15 percent. The shrinkage factor does not account for haul losses, because of the uncertainty associated with the various factors that contribute to haul losses.

We typically expect haul losses to be less than about 0.5 percent. Swell in the rock cut is dependent on a variety of factors that cannot be reliably predicted. The typical range of swell in rock cuts is about 20 to 50 percent.

## SPECIAL PROVISIONS

The project specifications should be written with special provisions addressing the following items.

1. Steel H-Piles
   a. The driving criteria for defining "practical refusal" will be developed by a wave equation analysis after the pile section and the driving equipment are known. At least three piles at the abutments should be monitored during installation using a dynamic analyzer to finalize the driving criteria.
   b. The contractor shall provide all applicable information regarding the hammer and other driving equipment for the geotechnical engineer's use in performing a wave equation analysis. This information must be supplied at least 15 working days in advance of actual pile installation. Any substitution of driving equipment might necessitate a similar time frame to reanalyze the substitution.

2. Drilled Piers
   a. Drilled piers shall have the minimum diameters and tip elevation as shown on the plans.
   b. Installation will require either advancing casing and/or slurry drilling techniques to maintain an open hole.
   c. Dewatering of piers should not be attempted unless casing is effectively seated into the claystone.
   d. Piers shall be promptly filled with concrete upon completion of drilling.
   e. Concrete should be placed using either a pump truck or a tremie, with the discharge kept at least 2 meters below the top of the fresh concrete to displace water and/or drilling slurry upward.

3. Earthwork – Imported fill material for raising the approach fills should meet the grading and plasticity requirements presented in the Earthwork section of this report.

## GENERAL COMMENTS

Terracon should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications.

The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include, either specifically or by implication, any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of David Evans & Associates for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either expressed or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.